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Mathematical Analysis for the Evaluation of Settlement and Load-Bearing Capacity of a Soil Base Adjacent to an Excavation Pit

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Abstract: The present paper states and provides an analytical solution for the problem of evaluating the settlement and load-bearing capacity of weighty soil layers of limited thickness resting upon incompressible soil bases and an excavation pit wall upon exposure of the foundation to a distributed load in the vicinity of a wall. The authors develop a method for determining the stressed state component in the reduced engineering problem based on the Ribere–Faylon trigonometric series and for accounting for the nonlinear deformation properties of soils, building on the analytical dependencies of S.S. Grigoryan and S.P. Timoshenko. In order to determine the relationship between stress and strain, the Hencky’s physical equation systems were used. They factor in the impact of average stresses σ_m on the shear modulus of elasticity $G(\sigma_m)$ and volumetric modulus of elasticity $K(\sigma_m)$. The obtained solutions make it possible to assess the deformation of soil bases and the load-bearing capacity with respect to nonlinear properties in a way that accurately corresponds to the actual performance of subsoils exposed to loading. The theoretical results are followed by numerical experiments to prove their validity.

Keywords: stress–strain state of subsoil; Ribere–Faylon trigonometric series; Hencky’s physical equations; volumetric and shear strains; foundation settlement; load-bearing capacity of subsoil; excavation pit wall



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1. Introduction

One of the critical problems in applied soil mechanics for high-rise construction involving substantial subsurfaces is quantitative assessment of the stress–strain state (SSS) of the soil massif adjacent to the deep excavation pit of the walls and subsurface structures and the factoring in of engineering–geological conditions. When the subsurface of a high-rise building interacts with the adjacent soil massif in the vicinity of the excavation pit walls and the foundation sublayer, a heterogeneous stress–strain state arises that is transformed in space—either during the construction stage or during the building’s operation. The main difficulties arise when the soil massif is heterogeneous and has elasto-plastic properties and when the additional load is applied in the proximity of an excavation pit.

The methods for forecasting the short-term and long-term settlements of foundations and soil bases were developed by such Russian scientists as N.A. Tsytoich [1], V.A. Florin [2], K.E. Egorov [3], N.M. Gersevanov [4], S.P. Timoshenko [5], Yu. K. Zaretsky [6], Z.G. Ter-Martirosyan [7], and others [8,9]. We can emphasize the work [10–23] where the methods for the analysis of foundation and soil-base settlement are described. Such analyses are based on a method of layered summation, without regard for changes in the deformation properties of soils, as a function of the stress level and thickness of the soil massif that do not correspond to the actual values of settlement in most cases. Due to this, methods for the quantitative assessment of the settlement and load-bearing capacity, factoring in the elasto-plastic properties of soils, of foundations and soil bases of finite width are gaining momentum and practical importance.

2. Materials and Methods

2.1. Theoretical Basis for Determination of the Settlement of a Weighty Soil Layer Resting upon an Incompressible Soil Base upon Exposure of the Foundation to a Distributed Load in the Vicinity of an Excavation Pit Wall

Consider the action of the distributed load $q = \text{const}$ on the horizontal section of width $b = 2a$ at a distance d from the edge of an envelope structure with a rectangular profile on a soil base resting on an incompressible soil layer. It is assumed that the vertical wall is fixed by struts, whereas vertical shifts in the soil are permitted (see Figure 2). It is known that, for the given problem, the components of the stressed state of the soil ground, being one quarter of a plane, can be determined using the Ribere–Faylon trigonometric series with the method developed by Z.G. Ter-Martirosyan in the following manner [30]:

$$\sigma_y(x, y) = \frac{qa}{l} + \frac{4q}{\pi} \sum_{m=1}^{\infty} \frac{\sin \frac{m\pi a}{l}}{m} \frac{\left[\left(\frac{m\pi h}{l} \text{ch} \frac{m\pi h}{l} + \text{sh} \frac{m\pi h}{l} \right) \text{ch} \frac{m\pi(y-h)}{l} - \frac{m\pi(y-h)}{l} \text{sh} \frac{m\pi(y-h)}{l} \text{sh} \frac{m\pi h}{l} \right]}{\text{sh} \frac{2m\pi h}{l} + \frac{2m\pi h}{l}} \cos \frac{m\pi x}{l} \quad (2)$$

$$\sigma_x(x, y) = \frac{qa}{l} \frac{\nu}{1-\nu} - \frac{4q}{\pi} \sum_{m=1}^{\infty} \frac{\sin \frac{m\pi a}{l}}{m} \frac{\left[\left(\frac{m\pi h}{l} \text{ch} \frac{m\pi h}{l} - \text{sh} \frac{m\pi h}{l} \right) \text{ch} \frac{m\pi(y-h)}{l} - \frac{m\pi(y-h)}{l} \text{sh} \frac{m\pi(y-h)}{l} \text{sh} \frac{m\pi h}{l} \right]}{\text{sh} \frac{2m\pi h}{l} + \frac{2m\pi h}{l}} \cos \frac{m\pi x}{l} \quad (3)$$

$$\tau_{xy}(x, y) = -\frac{4q}{\pi} \sum_{m=1}^{\infty} \frac{\sin \frac{m\pi a}{l}}{m} \frac{\left[\frac{m\pi h}{l} \text{ch} \frac{m\pi h}{l} \text{sh} \frac{m\pi(y-h)}{l} - \frac{m\pi(y-h)}{l} \text{ch} \frac{m\pi(y-h)}{l} \text{sh} \frac{m\pi h}{l} \right]}{\text{sh} \frac{2m\pi h}{l} + \frac{2m\pi h}{l}} \sin \frac{m\pi x}{l} \quad (4)$$

$$\sigma_m(x, y) = \frac{1+\nu}{3} \left[\frac{qa}{l} \frac{1}{1-\nu} + \frac{8q}{\pi} \sum_{m=1}^{\infty} \frac{\sin \frac{m\pi a}{l}}{m} \frac{\text{sh} \frac{m\pi a}{l} \text{ch} \frac{m\pi(y-h)}{l}}{\text{sh} \frac{2m\pi h}{l} + \frac{2m\pi h}{l}} \cos \frac{m\pi x}{l} \right] \quad (5)$$

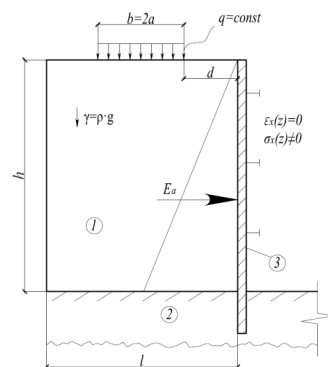


Figure 2. Design scheme for the interaction of a weighty layer (1) with thickness (h) resting upon an incompressible soil base (2) with a vertically fixed retaining wall (3) under a distributed load $q = \text{const}$ along the strip $b = 2a$ at a distance d from the retaining wall.

2.2. Hencky's Physical Equation System

The strains of the soil base ϵ_z were determined as the sum of the shear and volumetric strains ($\epsilon_z = \epsilon_{z\gamma} + \epsilon_{zv}$). Hencky's physical equation system [29], which allows the determination of the linear and nonlinear dependencies of stresses on strains, has the following form:

$$\epsilon_x = \chi(\sigma_x - \sigma_m) + \chi^* \cdot \sigma_m; \gamma_{xy} = 2\chi \cdot \tau_{xy} \quad (6)$$

$$\epsilon_y = \chi(\sigma_y - \sigma_m) + \chi^* \cdot \sigma_m; \gamma_{yz} = 2\chi \cdot \tau_{yz} \quad (7)$$

$$\epsilon_z = \chi(\sigma_z - \sigma_m) + \chi^* \cdot \sigma_m; \gamma_{zx} = 2\chi \cdot \tau_{zx} \quad (8)$$

where

$$\chi = \frac{\gamma_i}{2\tau_i} = \frac{f(\tau_i, \sigma_m, \mu_\sigma)}{2\tau_i}; \quad (9)$$

$$\chi^* = \frac{\varepsilon_m}{\sigma_m} = \frac{f^*(\tau_i, \sigma_m, \mu_\sigma)}{2\tau_i}, \quad (10)$$

Hencky's equations [29] at $\chi = 1/2G$ and $\chi^* = 1/K$, where $G = E/2(1 + \nu)$ and $K = E/(1 - 2\nu)$, can be graded into Hooke's equation system.

2.3. Analytical Models of Soil Bases

The dependency proposed by the academician of the Russian Academy of Sciences (RAS) S.S. Grigoryan was assumed as an analytical model for determining nonlinear volumetric strains [24]. It is as follows:

$$\varepsilon_m(\sigma_m) = \varepsilon^*(1 - e^{-\alpha\sigma_m}) \quad (11)$$

The tangent modulus of the volumetric strain K can be estimated by dividing the expression ε_m by σ_m ; i.e.,

$$\frac{\varepsilon_m}{\sigma_m} = \frac{1}{K} = \frac{\varepsilon^*(1 - e^{-\alpha\sigma_m})}{\sigma_m} \quad (12)$$

$$\frac{\varepsilon_m}{\sigma_m} = \frac{1}{K} = \frac{\varepsilon^*(1 - e^{-\alpha\sigma_m})}{\sigma_m} \quad (13)$$

at $\sigma_m \rightarrow \infty$; $\varepsilon_m \rightarrow \varepsilon^*$, but when $\alpha = 0$, $\varepsilon^* = \varepsilon_m$ and we obtain the linear dependency $K = \varepsilon_m/\sigma_m$.

To describe the elasto-plastic properties of cohesive soil exposed to shear loading, we can use the dependency proposed by S.P. Timoshenko [5], which has the following form with regard to soil ground:

$$\gamma_i = \frac{\tau_i}{G^0} \frac{\tau_i^*}{\tau_i^* - \tau_i} \quad (14)$$

where γ_i is the shearing strain intensity; τ_i are the acting values of the shearing stress intensity; τ_i^* are the ultimate values of the shearing stress; and G^0 is the shear modulus (elasticity modulus) for the initial part of the curve γ_i - τ_i .

$$\tau_i^* = (\sigma_m + \sigma_g) \cdot tg\varphi + c_i \quad (15)$$

where φ_i and c_i are the ultimate values of the strength parameters determined under the limit line of the dependency τ_i - σ_m and σ_g is the residual stress.

The secant shear modulus can be calculated with the following formula:

$$G = G^0(1 - \frac{\tau_i^*}{\tau_i^* - \tau_i}) \quad (16)$$

In the simplest case involving the linear dependency of stresses on strains with the parameters G and K , the settlement can be determined with an analytical solution for the axis z . Then, we can write:

$$\frac{\varepsilon_m}{\sigma_m} = \frac{1}{K} = \frac{\varepsilon^*(1 - e^{-\alpha\sigma_m})}{\sigma_m} \quad (17)$$

$$K = \frac{\sigma_m}{\varepsilon_m} \quad (18)$$

To account for the nonlinear strain of a soil layer in the calculation of its vertical displacements, Hencky's equation can be considered [29]:

$$\varepsilon_z = \frac{\sigma_z - \sigma_m}{G(\sigma_m, \tau_i/\tau_i^*)} + \frac{\sigma_m}{K}, \quad (19)$$

Inserting $G(\sigma_m, \tau_i)$ and $K(\sigma_m)$, we obtain nonlinear equations:

$$\varepsilon_{z,v} = \varepsilon^*(1 - e^{-\alpha\sigma_m}) \quad (20)$$

$$\varepsilon_{z,\gamma} = \frac{\sigma_z - \sigma_m}{2G_e(\sigma_m, \tau_i/\tau_i^*)}; \quad (21)$$

where $\tau_i = \frac{\sigma_1 - \sigma_3}{\sqrt{3}} = \frac{\sigma_z - \sigma_x}{\sqrt{3}}$ (at the axis of the distributed load center), $\tau_i^* = (\sigma_m + \sigma_g) \cdot \tan \varphi + c_i$, and σ_{zp} and σ_{xp} are determined using Equations (2)–(5).

3. Results

Calculation using Equations (2)–(5) in the software complex MathCAD made it possible to determine the components of stresses along the entire plane at $z > 0$ and $\pm x$ according to the computational scheme (see Figure 2). For the isopoles, the stress component for the values $q = 100$ kPa, $d_1 = 6$ m, and $b = 2a = 3$ m are presented in Figure 3. Furthermore, results were obtained for the values $q = 100$ kPa, $d_2 = 2$ m, and $b = 2a = 3$ m (see Figure 4). The given values were randomly selected based on the experience of designing similar structures in order to quantify the proposed solution.

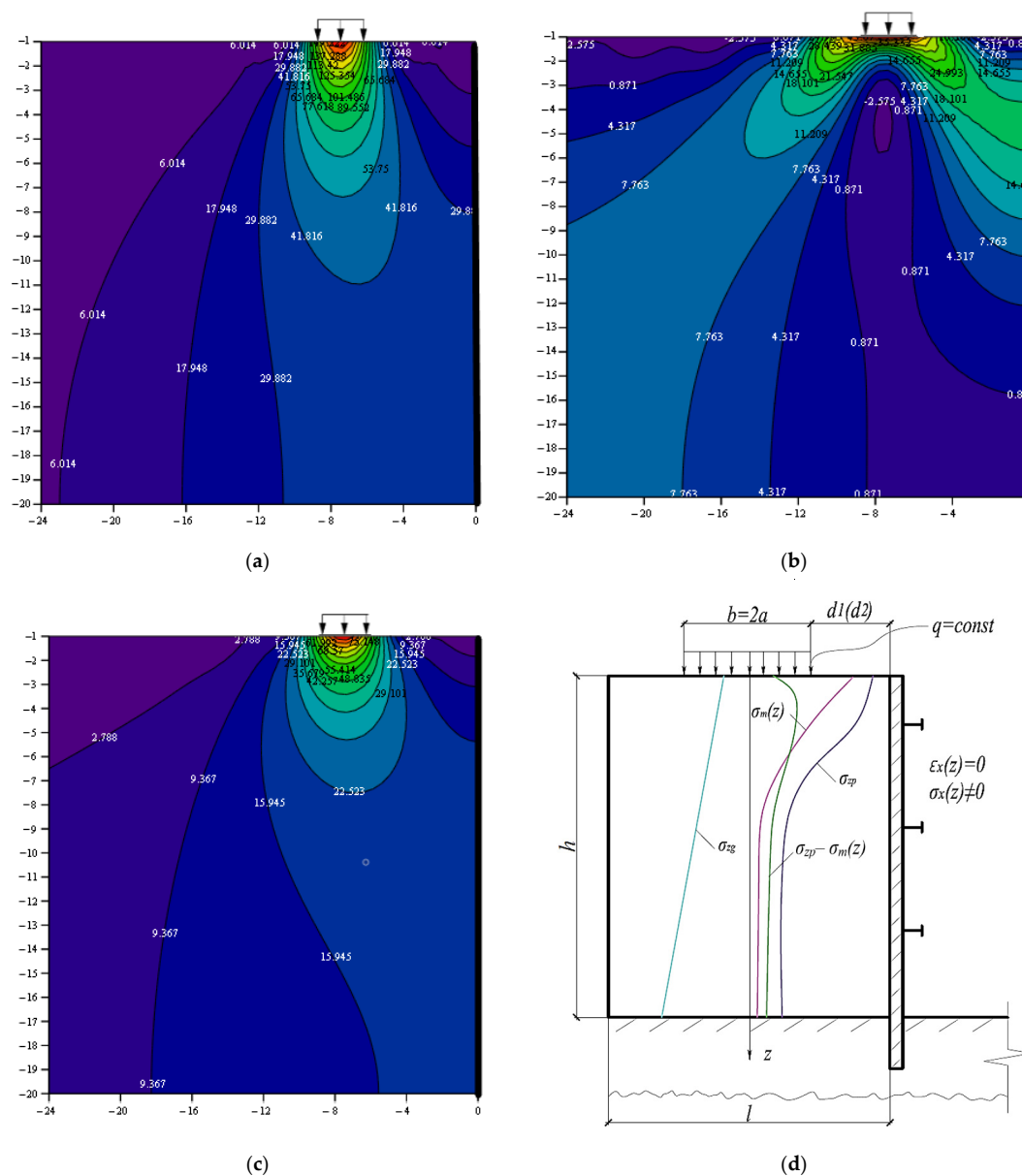


Figure 3. Stress isopoles at $q = 100$ kPa, $d_1 = 6$ m, and $b = 2a = 3$ m: (a) vertical stress σ_y ; (b) horizontal stress σ_x ; (c) mean stresses σ_m ; (d) design scheme for determination of the shear and volumetric strains of a weighty soil of limited thickness resting upon an incompressible soil base based on Hencky's physical equations.

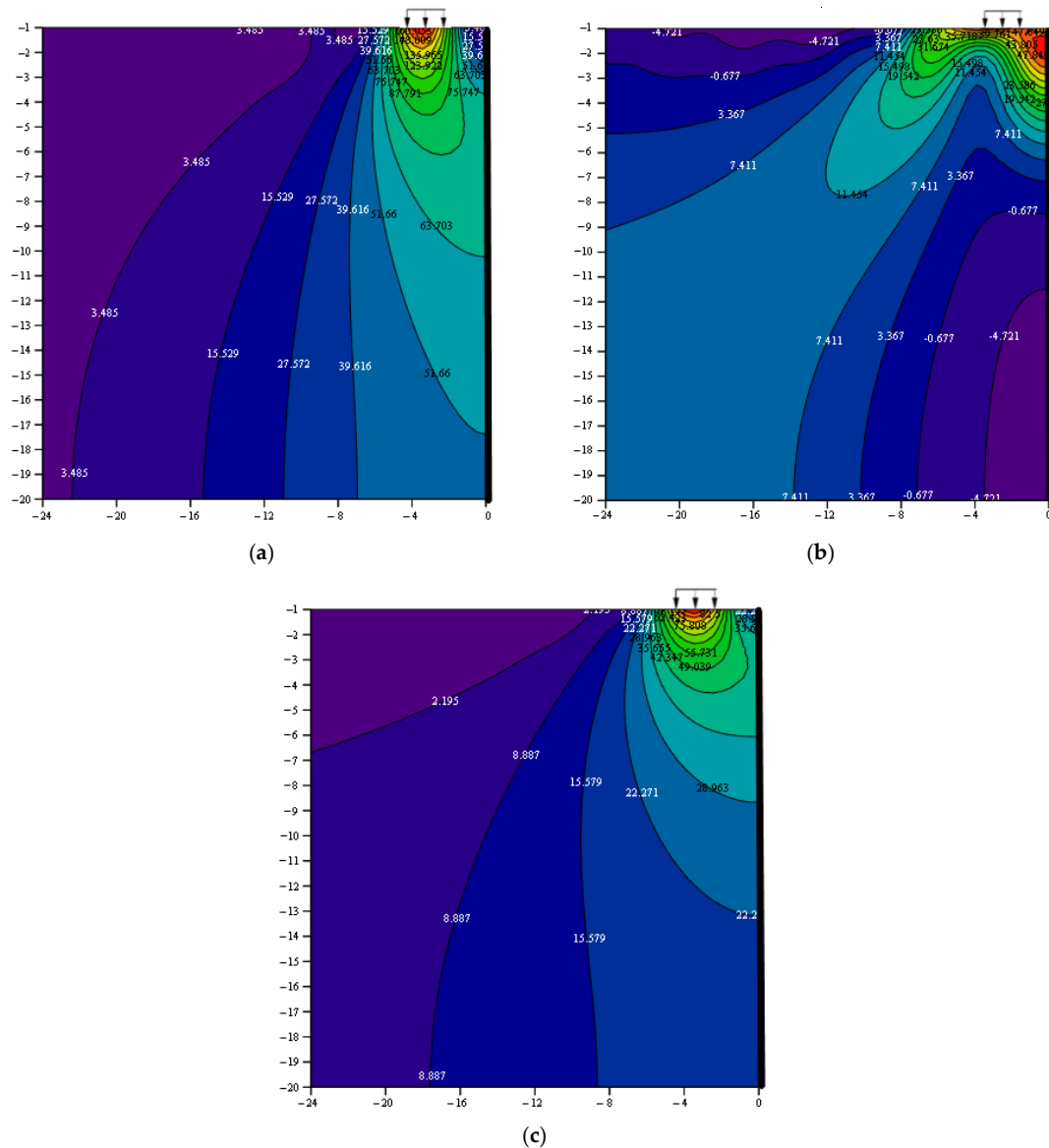


Figure 4. Stress isopoles at $q = 100$ kPa, $d_2 = 2$ m, and $b = 2a = 3$ m: (a) vertical stress σ_y ; (b) horizontal stress σ_x ; (c) mean stresses σ_m .

The settlement curves were drawn for the considered problem (see Figures 3 and 4) with allowances made for the elasto-plastic properties of the soil base at various distance d (d_1, d_2) from the fence wall to the distributed load. The total deformation of a soil base with a thickness of $h = 20$ m at various values where $x \geq 0$ can be calculated by means of the following integrals:

$$S = \int_0^h \varepsilon_{z,vi}(dz) + \int_0^h \varepsilon_{z,\gamma i}(dz) = S_v + S_\gamma \quad (22)$$

Having analyzed Equations (20) and (21), it was concluded that the constituent of the volumetric strain $\varepsilon_{z,\gamma}$ would be of the decaying type with a growth of σ_z and when $\sigma_m \rightarrow \infty$; $\varepsilon_{z,\gamma} \rightarrow \varepsilon^*$. At the same time, with the increase in σ_z , the value of $\varepsilon_{z,\gamma}$ would be initially of the linear type and then would progress to the intensive growth stage, as at $\tau_i \rightarrow \tau_i^*$; $\varepsilon_{z,\gamma} \rightarrow \infty$. Therefore, the total value for the strain would have a double curvature; i.e., at the initial stage at $\tau_i < \tau_i^*$ $\varepsilon_{z,\gamma}$ it would be of the decaying type and then at $\tau_i \rightarrow \tau_i^*$ it would change to the stage of progressive deformation.

For comparison of settlement paths, we assumed the following distances from the fence wall to the point of application of the distributed load: $d_1 = 6$ m and $d_2 = 2$ m. The analysis of the total strain arising in a soil base $\varepsilon_z = \varepsilon_{z,v} + \varepsilon_{z,\gamma}$ along the different vertical lines was implemented for the center of the distributed load for each scenario ($x_1 = -7.5$; $x_2 = -3.5$). The parameters of the mechanical properties of the soils were assumed to be $\varepsilon^* = 0.003$, $\alpha = 0.4$, $\nu = 0.35$, $G_e = 7000$ kPa, $\varphi = 25^\circ$, and $c = 14$ kPa. The results are presented in Figures 5 and 6.

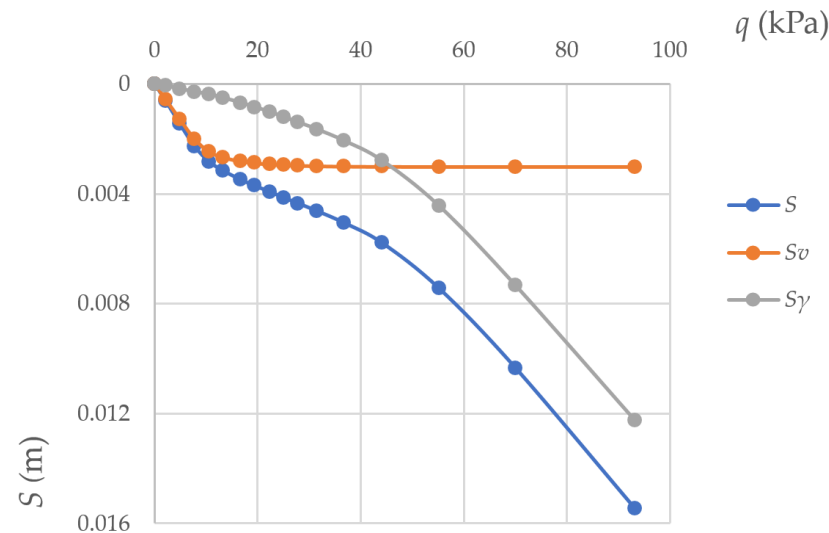


Figure 5. The dependency curves for S_γ , S_v , and S calculated using Equations (20) and (21) and the load q at $d_1 = 6$ m when the distance between the fence wall and the point of application of the distributed load is $d_1 = 6$ m.

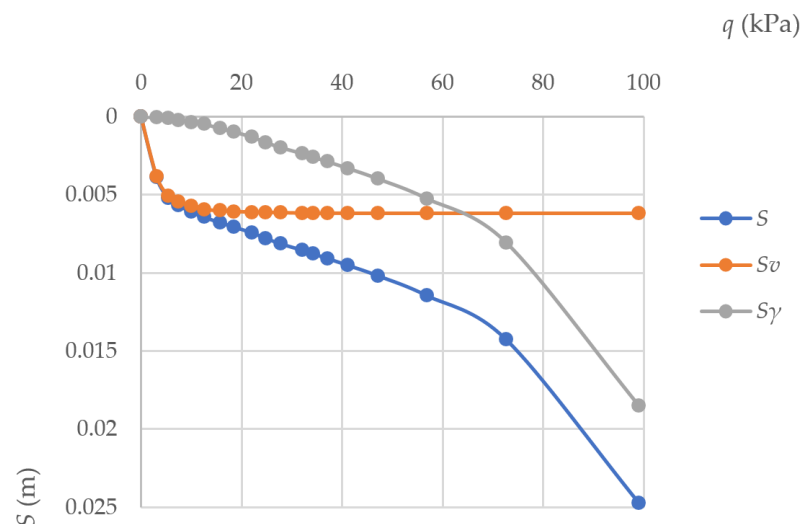


Figure 6. The dependency curves for S_γ , S_v , and S calculated using Equations (20) and (21) and the load q at $d_2 = 2$ m when the distance between the fence wall and the point of application of the distributed load is $d_2 = 2$ m.

Juxtaposing the total settlement of the soil base with various distances between the fence wall and the point of application of the distributed load, we obtained the combined dependency graph $S - q$ (see Figure 7).

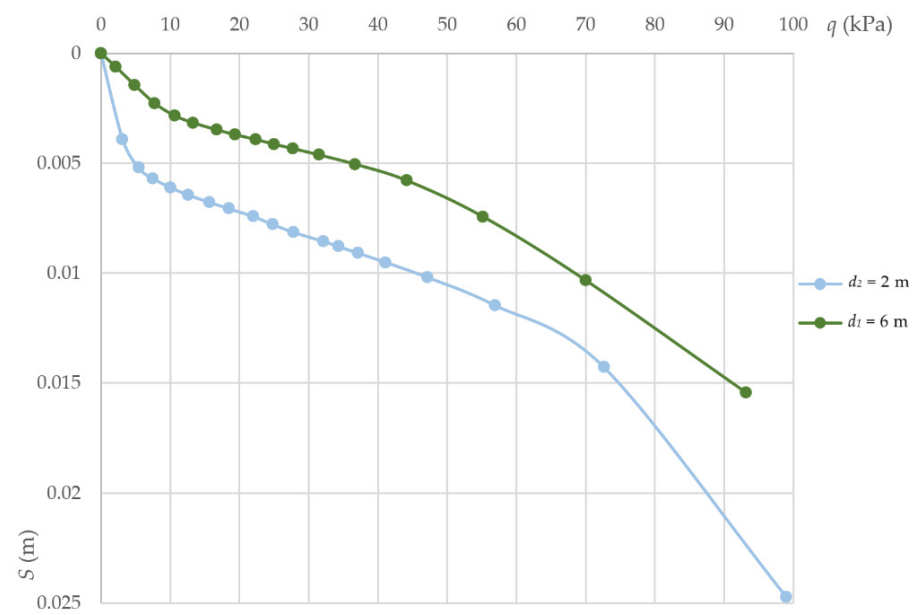


Figure 7. The dependency curves for the total settlement S of the load q calculated with Equations (20) and (21) with various distances $d_1 = 6$ m and $d_2 = 2$ m between the fence wall and the point of application of the distributed load.

We also obtained the diagrams of the horizontal strains ε_x and vertical strains ε_y on the verticals with various distances $d_1 = 6$ m and $d_2 = 2$ m between the fence wall and the distributed load. In Figures 8 and 9, a substantial difference in the total horizontal (a factor of 1.3) and vertical (twofold) strains of the right edge of a foundation with a distributed load $q = 100$ kPa with various distances d between the fence wall and the distributed load can be seen.

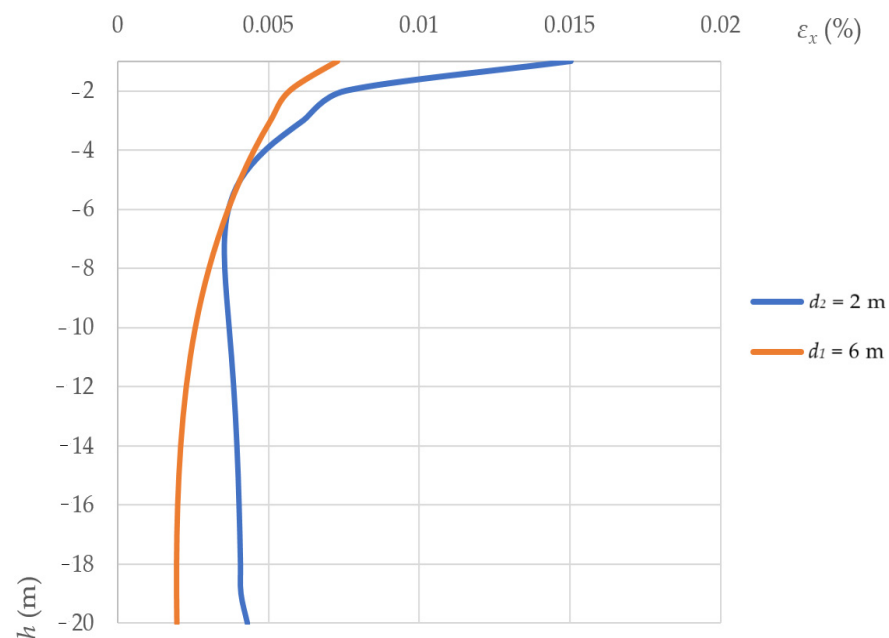


Figure 8. Diagram of horizontal strains ε_x along the vertical line at distances $d_1 = 6$ m and $d_2 = 2$ m.

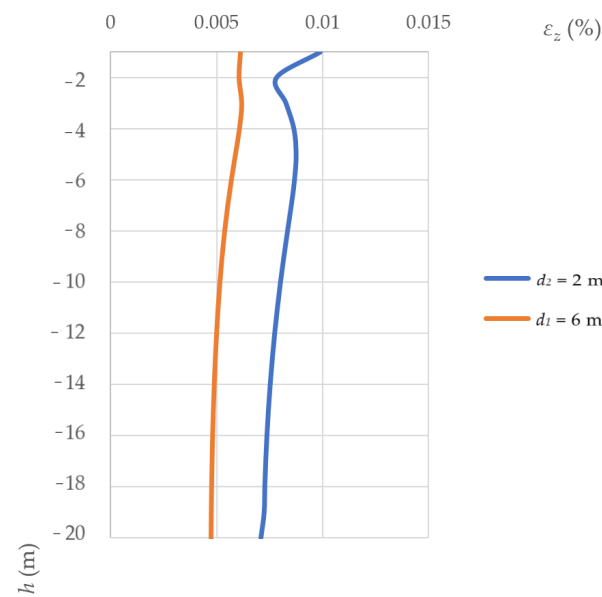


Figure 9. Diagrams of vertical strains ε_y along the vertical line at distances $d_1 = 6$ m and $d_2 = 2$ m.

4. Discussion

From the analysis of the outlined results, it follows that the smaller the distance between the fence wall and the distributed load, the higher the value of the total strain is.

It is evident that, depending on the various parameters of the layers' deformability, the total settlement of the soil base in general changes across a broad range from a decaying to progressive settlement (see Figure 10). The dependency curves $S - q$ were obtained for the scenario where the distance between the fence wall and the distributed load is $d_2 = 2$ m.

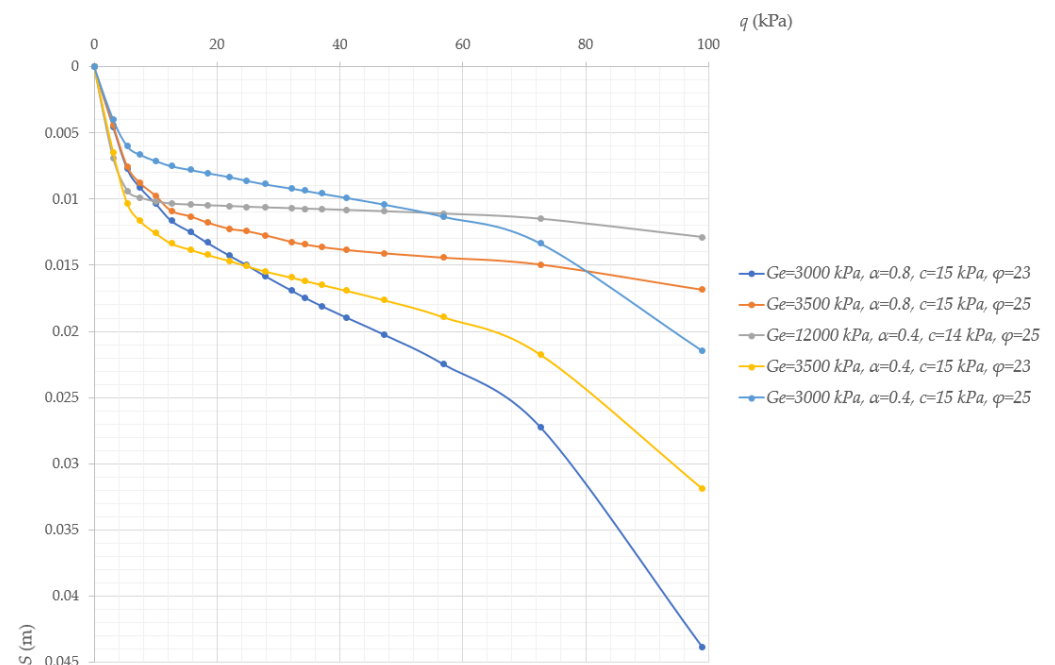


Figure 10. Dependency curves $S - q$ with a distance $d_2 = 2$ m between the fence wall and the distributed load drawn using the summability method with various parameters of deformability (G_e , α and c) and strength (φ and ψ).

Figures 5–10 show that the presented analytical method for the calculation of the settlement and load-bearing capacity of a soil base exposed to distributed loading at various distance from the edge of a excavation pit, implemented by means of the computational

models of S.P. Timoshenko and S.S. Grigoryan, along with numerical methods, makes it possible to factor in the nonlinear character of soil behavior to the fullest extent and with a reasonable degree of confidence. Moreover, when designing excavation pits, it is necessary to take into account the distance from the edges to adjacent buildings and structures located within the influence zone of the new construction.

5. Conclusions

Summing up the obtained results, the following can be concluded:

- The selected geomechanical soil base model (its geometric parameters and initial and boundary conditions), as well as the computational model of the soil ground (linear, nonlinear, and rheological) and the type of physical equations used (Hooke's system and Hencky's system), have a significant impact on the type of settlement–load curve ($S - q$) and also on the load-bearing capacity of the soil ground.
- The computational model applied in the present work, along with the elasto-plastic model for the shear strain and the nonlinear model for the volumetric strain in Hencky's physical equation system, enabled us to present the linear soil deformation ε (σ, τ) in the form of the sum of the volumetric and shear constituents of this linear deformation ($\varepsilon_z = \varepsilon_{z,N} + \varepsilon_{z,\gamma}$). In this case, the strain–stress curve ($\varepsilon_z - \sigma_z$) could develop along both the decaying path and progressive path (double-curved graph).
- The implemented strain analysis showed that, in the vicinity of the vertical excavation pit exposed to distributed load, the total foundation settlement was 1.6 times higher at the distance $d_2 = 2$ m than at $d_1 = 6$ m with the values for the deformation parameters of soils used in the calculation in this article. We also revealed a substantial difference in the total horizontal (a factor of 1.3) and vertical (twofold) strains at the right edge of the foundation exposed to a distributed load $q = 100$ kPa with various distances d between the fence wall and the distributed load. Thus, when designing excavation pits, it is necessary to take into account the distance from the edges to adjacent buildings and structures located within the influence zone of the new construction. This is done to avoid the development of strains in the foundation and soil base above the designed level.
- The analysis of existing methods describing the stress–strain state, as well as the calculation of the settlement and load-bearing capacity of soil bases exposed to additional loading—in particular, in the vicinity of excavation pits—showed that there are currently no analytical methods accounting for the elasto-plastic properties of soils. The presented analytical method for the calculation of the settlement and load-bearing capacity of soil bases exposed to distributed loading at various distances from the edge of an excavation pit, implemented by means of the computational models of S.P. Timoshenko and S.S. Grigoryan, along with numerical methods, makes it possible to factor in the nonlinear character of soil behavior to the fullest extent. It also makes it possible to predict the settlement and load-bearing capacity with a reasonable degree of confidence.

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